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Calibration of the Ontario Highway Bridge Design Code

Andrej S. Nowak
Department of Civil Engineering
University of Waterloo

Akhilesh C. Agarwal
Project Research Officer
Research and Development Branch
Ministry of Transportation and Communications

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ABSTRACT

This report describes the procedure and results of the calibration of the Ontario Highway Bridge Design Code. It covers calculation of six load factors (three different types of dead load, live load, impact and secondary effects of prestressing in continuous structures) and it deals with performance factors for composite steel girders, pretensioned concrete girders, and post-tensioned concrete decks.

The objective of the study was to select such load and performance factors as would provide bridges designed according to the new code with a prescribed safety level.

Nineteen bridges recently built in Ontario were selected. Proportions of load components calculated for these bridges were assumed as typical for future structures.

Safety levels, in terms of safety indices, were computed for the selected bridges. Then load and performance factors were calculated aiming to achieve safety indices close to the selected target value.

The resistances required by the code with new load and performance factors were compared to the resistances of existing bridges. In many cases the new requirements are considerably less.

1/ INTRODUCTION

The Ontario Ministry of Transportation and Communications (MTC) is developing a new code for the design of highway bridges. The Ontario Highway Bridge Design Code (OHBD Code) [1], published in 1979, is written in a "Level I" reliability format using load and resistance factors, similar to the Limit States Design option available in the National Building Code of Canada since 1975.

A practical Level I code is a compromise between three requirements: reliability, legal clarity, and operational simplicity. The aim is to simultaneously:

(a) provide for the prescribed safety level in all design situations covered by the code,
(b) keep the jurisdictional area of the code at a convenient size, and
(c) have a simple format enabling easy use of the code.

This report describes mainly how the reliability requirement given in item (a) is met under the practical restraints of format dictated by (b) and (c). It deals, in particular, with the calibration of the code, that is, the selection of values of load and performance factors so as to provide for the prescribed safety level. The procedure of calibration that was developed is based on the studies described in References 2, 3 and 4. It was assumed that design parameters such as values of load effects and resistances were random variables. The random variation was considered as a result of inaccuracies in con-struction, use, the analytical model, prediction of capacities of sections, prediction of load model, and load values in the lifetime of the bridge. However, this study does not consider the variation of design parameters due to gross errors; for example, negligence leading to the use of wrong materials or calculation errors

The subjects of calibration were load factors and performance factors. The load factors were:

α_1 = load factor associated with dead load component D_1, constituting the weight of factory-made main structural members;

α_2 = load factor associated with dead load component D_2, constituting the weight of concrete cast-in-situ and all the remaining superstructure, except asphalt;

α_3 = load factor associated with dead load component D_3, which is the weight of asphalt surface;

α_L = load factor associated with live load effect L, which is the static effect of trucks moving on the bridge;

α_I = load factor associated with impact load effect I, which is the dynamic effect of trucks moving on the bridge; and

α_{P_s} = load factor associated with the secondary prestressing effect P_s in continuous structures.

The performance factor is associated with the resistance R. The basic design equation involving the above mentioned load components is:

$$\phi R = \alpha_1 D_1 + \alpha_2 D_2 + \alpha_3 D_3 + \alpha_L L + \alpha_I I + \alpha_{P_s} P_s \qquad (1)$$

The calibration was performed for the ultimate limit state prescribed by the OHBD Code.

2/ SAFETY MEASURE

The available methods of structural safety checking are, like code formats, classified into levels [2]. In the present study "Level II" rationales were used. They are convenient for evaluation of the sets of partial safety factors in Level I codes. In Level II methods, the degree of safety is characterised by a safety index, β, or an equivalent operational measure of probability, and not directly in terms of probability.

A second moment safety index is defined, in general, as:

$$\beta = \frac{\overline{Z}}{\sigma_Z} \qquad (2)$$

where:
$Z = g(X_1, \ldots, X_n)$ is the formulation variable corresponding to the failure criterion, $g(X_1,\ldots,X_n) < 0$, X_i are the basic random variables in the formulation of the problem, and $\overline{Z}$ and σ_Z are statistics of Z, namely the mean and the standard deviation, respectively.

The various Level II methods available use different formulation variables. A commonly used safety index is that suggested by Rosenblueth and Esteva [5]. The failure criterion is taken in the form:

$$Z = \ell n\,(R/S) = \ell n(R) - \ell n(S) = 0 \qquad (3)$$

where R is resistance and S is load effect. In practice, the mean and standard deviation of Z, Equation 3, can be taken as the first order approximations as follows [6].

$$\overline{Z} = \ell n(\overline{R}) - \ell n(\overline{S}) \qquad (4)$$

$$\sigma_Z = \sqrt{V_R^2 + V_S^2} \qquad (5)$$

where $\bar{R}$, V_R are the mean and coefficient of the variation of resistance, and $\bar{S}$, V_S are the mean and coefficient of variation of load effect, respectively. Hence, the safety index is:

$$\beta = \frac{\ln \bar{R}/\bar{S}}{\sqrt{V_R^2 + V_S^2}} \qquad (6)$$

In this formula for β, the effects of all load components are lumped into a single random variable S, and all components of resistance are lumped into a single random variable R. There are other safety indices that produce a more accurate estimation of the probability of failure under special circumstances [2]. Safety indices calculated for a particular design situation using different formulae can generally be different. However, if the same formula is used in different design situations, then the results are comparable. The latter applies in the present code calibration. Safety indices are used to compare safety level in various design situations, and simple (R,S) formats, especially Equation 6, have proved to be practical.

3/ OBJECTIVE IN CALIBRATION

The objective of calibration was to select a set of values for the load factors α_i to be used in all design situations, and a set of performance factors, one for each technology and limit state (e.g., flexure of composite steel girders), so as to provide for the required safety level in design situations covered by the code. The target safety level was expressed in terms of the Rosenblueth-Esteva safety index [5] given by Equation 6, taken as $\beta_T = 3.5$ [1].

It is practically impossible to develop a code such that all structures would have a safety index exactly equal to the target value. Such a code would require far too many different load and performance factors, and a complex procedure for evaluating the structural safety index from the component safety indices. In the present calibration, safety was checked for selected design situations considered as typical in bridge design. The choice of the set of code parameters, α_i and ϕ, was made on the basis of closeness to the target safety index.

4/ SELECTION OF TYPICAL DESIGN SITUATIONS

For the purpose of safety checking, 19 bridges constructed recently in Ontario were selected. It was assumed that they represented the most common structures in the province and were typical for the application of the OHBD Code.

For each bridge several characteristic sections were selected, and internal forces (moments and shears) due to various load components were calculated. It was further assumed that the proportions of load component effects found in these sections, (i.e. rtios $M_1 : M_2 : M_3 : M_L : M_I : M_{P_s}$ and $Q_1 : Q_2 : Q_3 : Q_L : Q_I : Q_{P_s}$, L, where M_i are moments and Q_i are shear forces due to D_1, D_2, D_3, I and P_i respectively), represent typical design situations covered by the code. The proportions were found to be rather insensitive to changes of the code parameters α_i and ϕ, and in calibration their variation was neglected.

The process of calibration was based entirely on safety checks for the selected proportions of load component effects. Ultimate capacities (resistances) of the sections were calculated also in order to evaluate the safety indices for the existing bridges. The selected categories of bridges and their spans are listed in Table 1.

5/ RESISTANCE OF BRIDGE SECTIONS

In second moment methods of safety checking, a random variable is characterized by two values, such as the mean and the coefficient of variation. In the OHBD code calibration, resistances were characterised conveniently by their nominal values, the ratios of mean to nominal values, λ_R, and the coefficients of variation, V_R. The uncertainty in the resistance was decomposed into three factors:

(1) uncertainty in material strength (M),
(2) uncertainty in fabrication (F), and
(3) uncertainty in strength analysis (P).

Ratios of mean to nominal resistances were considered for the three factors M, F, and P. Then λ_R was calculated as a product:

$$\lambda_R = \lambda_M \lambda_F \lambda_P . \qquad (7)$$

Similarly, the coefficients of variation of the resistance, V_R, were calculated from the coefficients of variation of the three components of uncertainty as:

$$V_R = \sqrt{V_M^2 + V_F^2 + V_P^2} \qquad (8)$$

The selection of λ_R and V_R was based on the following sources: composite steel and concrete bridges [7,8,9] and prestressed concrete bridges [11,12]. The adopted values are listed in Table 2.

6/ LOAD EFFECTS

The total load effect S is decomposed as the product of two factors [13]:

$$S = T\,E \tag{9}$$

where T is the loading with mean value $\overline{T} = \overline{D}_1 + \overline{D}_2 + \overline{D}_3 + \overline{L} + \overline{I} + \overline{P}_s$ and coefficient of variation V_T, and E is the influence factor with the mean value $\overline{E} = 1$ and coefficient of variation V_E =4% for single-span bridges and 6% for multi-span ones. The mean load effect S is:

$$\overline{S} = \overline{T}\,\overline{E} = \overline{D}_1 + \overline{D}_2 + \overline{D}_3 + \overline{L} + \overline{I} + \overline{P}_s \tag{10}$$

and the coefficient of variation, V_S, of S is:

$$V_S = \sqrt{\left(\frac{\sigma_T}{\overline{T}}\right)^2} \tag{11}$$

$$\sigma_T^2 = \sigma_{D_1}^2 + \sigma_{D_2}^2 + \sigma_{D_3}^2 + \sigma_L^2 + \sigma_I^2 + \sigma_{P_S}^2 = (V_1\,\overline{D}_1)^2 + (V_2\,\overline{D}_2)^2$$

$$+ (V_3\,\overline{D}_3)^2 + (V_L\,\overline{L})^2 + (V_I\,\overline{I})^2 + (V_{P_s}\,\overline{P}_s)^2 \tag{12}$$

where:

V_1, V_2, V_3, V_L V_I, V_{P_s} are the coefficients of variation of D_1, D_2, D_3, L, I and P_s, respectively.

The statistics (ratios of mean to nominal and coefficients of variation) of dead loads were assumed as independent of technology, span length, or limit state considered.

The means and coefficients of variation of the live loading were calculated using the statistics from the heavy vehicle survey in Ontario, carried out by MTC in 1975. The survey trucks were run through the selected influence lines for the selected 19 bridges so that the distributions of response were obtained for each of the considered limit states (moments and shears). Then, a negative exponential fit was tried for the tail of the response distribution and the mean largest response (L) was predicted for 50 years lifetime. The procedure and results were presented in Reference 14.

Design values of live loading were calculated according to provisions of the OHBD Code, i.e., the load was taken as the effect of one Ontario Highway Bridge Design (OHBD) truck, or 70% of the OHBD truck effect, plus a uniformly-distributed load of 12 kN/m, whichever gave a larger effect.

The values of mean and nominal effects of live load are plotted in Figure 1. They are given as fractions of the mean total load S, as given by Equation 10. The average ratio of mean to nominal effect was found equal to 1.12. However, for the midspan moments of 18 m span bridges, the ratio equals 1.50. The points corresponding to those short-span bridges are shown in a circle in Figure 1.

The uncertainty in live load was decomposed into a product of two factors: uncertainty of the modelled load with coefficient of variation 5%, and uncertainty of the load model with coefficient of variation 5%. The resulting coefficient of variation of live load, used in calibration, was V_L = 7.1%.

The calculations of the impact load effect were performed according to Reference 15. It was assumed that the mean impact effect, I, associated with the mean largest live load effect, L, is a fraction of L.

$$\bar{I} = (\Delta_1 I)\bar{L} \qquad (13)$$

where $\Delta_1 I$ is the dynamic increment for mean impact taken as a function of frequency of the bridge (see Figure 2).

The design values of impact effect were calculated according to the OHBD Code provisions as a fraction of nominal live load effect:

$$I = (\Delta_2 I)L \tag{14}$$

where $\Delta_2 I$ is the dynamic increment for nominal impact effect, given as a function of frequency of the bridge in Figure 2.

Coefficient of variation for impact load was assumed to be: V_I=45% [15].

In the case of multispan prestressed bridges, the effect of the prestressing force, P_s, was included. The ratio of mean to nominal value of P_s was assumed $\lambda_{P_s} = 0.98$, and coefficient of variation V_{P_s}= 3.6% [11, 12].

The statistics of load components used in calibration are given in Table 3.

7/ SAFETY INDICES FOR THE SELECTED EXISTING BRIDGES

For all selected bridges and limit states the resistances, R, (ultimate capacities) were calculated then safety indices were found using Equation 6. Figure 3 presents these safety indices versus $\ell n(\Theta) = \ell n(R/S)$. All points in Figure 3 fall in the area between two lines corresponding to coefficients of variation of 9% and 20%.

The resulting values of β show a considerable scatter, from 1.8 to over 14. Even safety indices corresponding to various limit states of the same bridge are often quite different. This could be partly explained by the fact that bridges were usually designed for the most unfavourable limit state and then the same depth or shape of section was used for all the structure for economical or aesthetic reasons. For each bridge, a point in Figure 3 corresponding to the lowest value of β for this bridge is denoted by an open symbol.

8/ LOAD FACTORS

It was assumed that factored values of each load component (i.e., design value multiplied by load factor) should have the same probability of being exceeded during the lifetime of the bridge., All load components, (e.g. X_i), are given by their mean, $\bar{X}_i$, coefficient of variation V_i and nominal (design) value X_i in Figure 4. Assuming that all distributions are normal, one can express the probability, P_n, of the factored values being exceeded by n, the distance from the mean in terms of the standard deviation, σ_i. Table 4 gives the relation between n and P_n for normal distribution. The load factors were calculated as a function of n as follows.

From Figure 4:

$$\alpha_i X_i = \bar{X}_i + n\sigma_i, \tag{15}$$

hence:

$$\alpha_i = \bar{X}_i / X_i (1 + n_{V_i}), \tag{16}$$

or:

$$\alpha_i = \lambda_i (1 + n_{V_i}). \tag{17}$$

Values of load factors as functions of n are plotted in Figure 5. The final selection of a set of load factors was made in accordance with the selection of ϕ factors.

9/ PERFORMANCE FACTORS

The criterion in selection of load and performance factors was closeness to the target value of the safety index, β_T. The performance factors, ϕ, were calculated in two steps. First, various sets of load factors were considered, corresponding to different n's. For each set of α's the values of ϕ's were calculated to provide for the target safety index $\beta_T = 3.5$. In Equation 6, replacing β by β_T and $\bar{R}$ by $\lambda_R R$, after simple transformations, the following expression for the required resistance, R, is derived:

$$R = \frac{1}{\lambda_R} \bar{S} \exp \left(\beta_T \sqrt{V_R^2 + V_S^2} \right) \qquad (18)$$

For this value of nominal resistance the resulting safety index is equal to β_T.

Replacing R in Equation 1 by Equation 18, after simple transformation, the expression for optimum ϕ results:

$$\phi = \lambda_R \frac{S_n}{\bar{S}} \exp \left(- \beta_T \sqrt{V_R^2 + V_S^2} \right) \qquad (19)$$

where $S_n = \alpha_1 D_1 + \alpha_2 D_2 + \alpha_3 D_3 + \alpha_L L + \alpha_I I + \alpha_{P_S} P_S$

The next step was to select ϕ values for various groups of bridges and limit states and check the resulting safety indices.

Both α's and ϕ's were rounded off to the nearest 0.05. The calibration gave the following code parameters:

(1) load factors:

$\alpha_1 = 1.10$

$\alpha_2 = 1.20$

$\alpha_3 = 1.50$

$\alpha_L = \alpha_I = 1.25$

$\alpha_{P_S} = 1.05$ for positive P_S,

$= 0.90$ for negative P_S

(2) Performance Factors:

- Steel Girders

  Moments: $\phi_1 = 0.90$

  Shears: $\phi_1 = 0.90$

- Pretensioned Girders

  Positive Moments: $\phi_2 = 0.85$

  Negative Moments: $\phi_3 = 0.90$

  Shears: $\phi_4 = 0.75$

- Post-tensioned Decks

  Moments: $\phi_5 = 0.85$

  Shears: $\phi_6 = 0.70$

Using the above values, safety indices β were calculated, and are displayed in Figure 6 as a function of $\theta(=R/S)$.

It is desirable to keep the number of different ϕ factors small to enable an easy use of the code. However, this creates difficulty in closely meeting the target value of the safety index. As an example, Figure 7 displays resulting safety indices for:

$$\phi_1 = 0.90$$
$$\phi_2 = \phi_3 = \phi_5 = 0.85$$
$$\phi_4 = \phi_6 = 0.70$$

To compare the new code requirements with the old ones, the ratios of the minimum resistances required by the new code and the resistances of the existing bridges were calculated. The values were plotted vs. β values for existing bridges in Figure 8. Comparing the resistances, one has to take into account that resistances of existing bridges include other influences such as rounding up by the designer, or utilization of a spare or used oversize structural component.

10/ CONCLUSIONS

The presented calibration procedure allows for a more uniform distribution of safety reserve in bridges to be designed according to the developed code. The bridges currently built in Ontario feature a rather considerable scatter of reliability measured in terms of safety index. After calibration, in most cases, the required resistances of bridge sections are smaller than those of existing structures. Nevertheless, in some cases of shear in pretensioned girders, the new requirements are more conservative.

11/ ACKNOWLEDGEMENTS

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TABLE 1/ CATEGORIES OF SELECTED BRIDGES

Category	Number of Bridges Considered	Spans (m)		
		Simply supported span(s)	Continuous	
			Longer Span(s)	Shorter Span(s)
Composite Steel Girders	6	18-42	21-78	12-42
Pretensioned Concrete AASHTO and CPCI Beams	7	18-33	24-32.4	12-32.1
Post-tensioned Concrete Decks	6	-	21-54	15-39.6

TABLE 2/ STATISTICS OF RESISTANCES

Category of bridge and limit state	Ratio of mean to nominal resistance, λ_R	Coefficient of variation of resistance, V_R (%)
Composite Steel and Concrete Sections		
positive moments		
-tension	1.16	10.4
-compression	1.10	12.6
negative moments	1.16	10.1
-shear	1.16	10.1
Pretensioned Girders		
-flexure	1.07	6.3
-shear	1.15	16.5
Post-tensioned Decks		
-flexure	1.07	6.6
-shear	1.15	16.5

TABLE 3/ STATISTICS OF LOAD COMPONENTS

Load Component	Ratio of mean to nominal	Coefficient of variation (%)
D1	1.03	4
D2	1.05	8
D3	1.10	20
L	varies, on average 1.12	7.1
I	varies, on average 0.66	45
PS	0.98	3.6

TABLE 4/ RELATION BETWEEN n AND Pn FOR NORMAL DISTRIBUTION

n	1.0	1.5	2.0	2.5	3.0	3.5
Pn	0.1587	0.0668	0 0228	0.0062	0.0013	0.0002

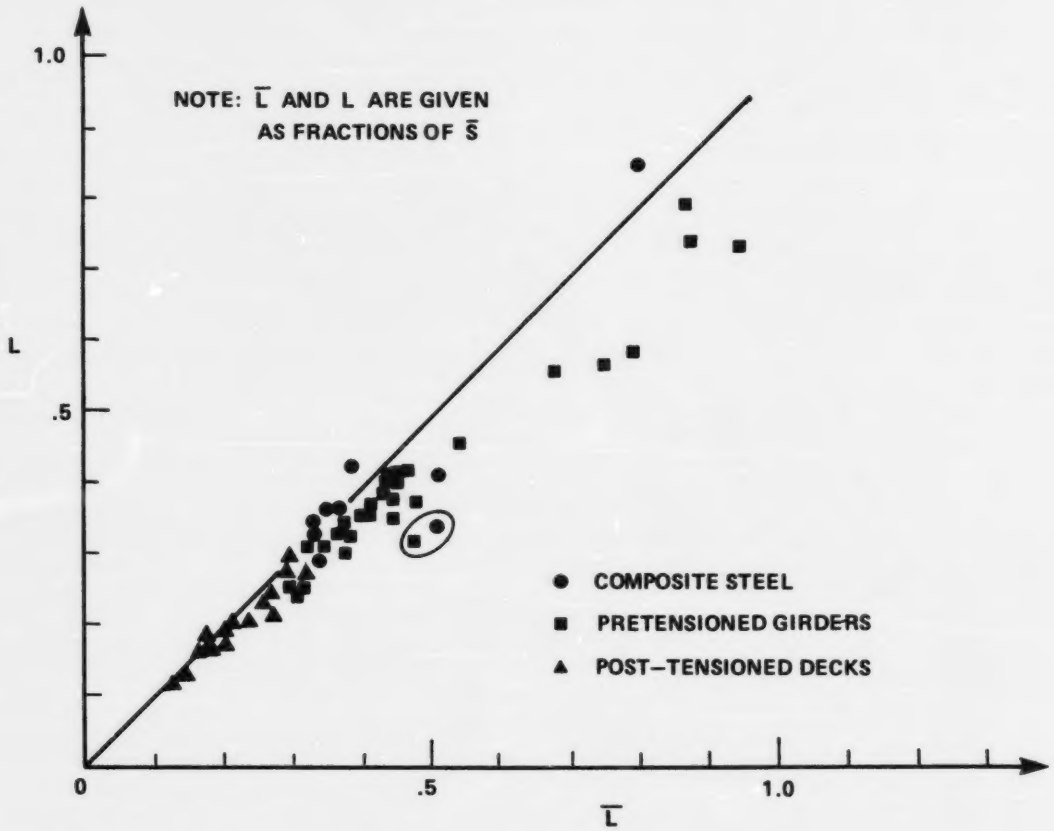

Figure 1/ A Comparison of Mean and Nominal Live Load Effects for Selected Bridges

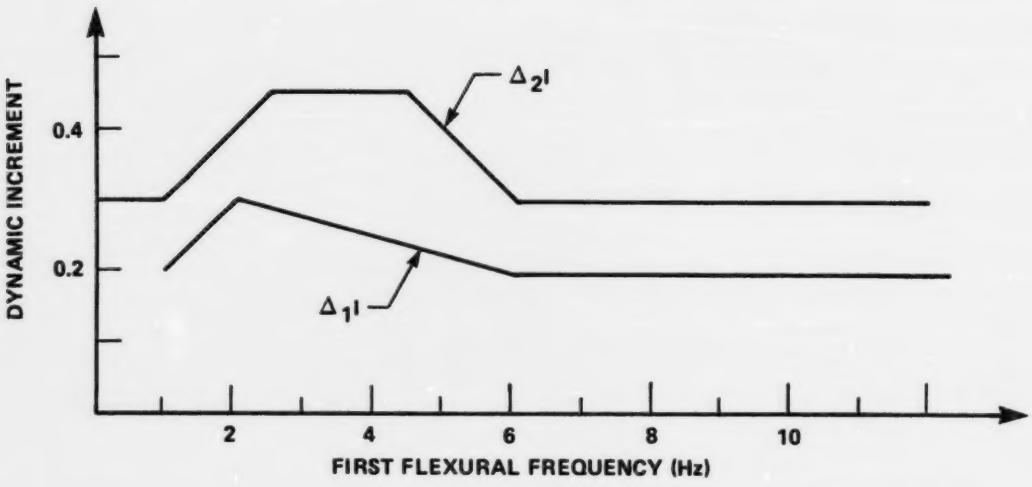

Figure 2/ Dynamic Increment for Mean Impact and the Dynamic Load Component

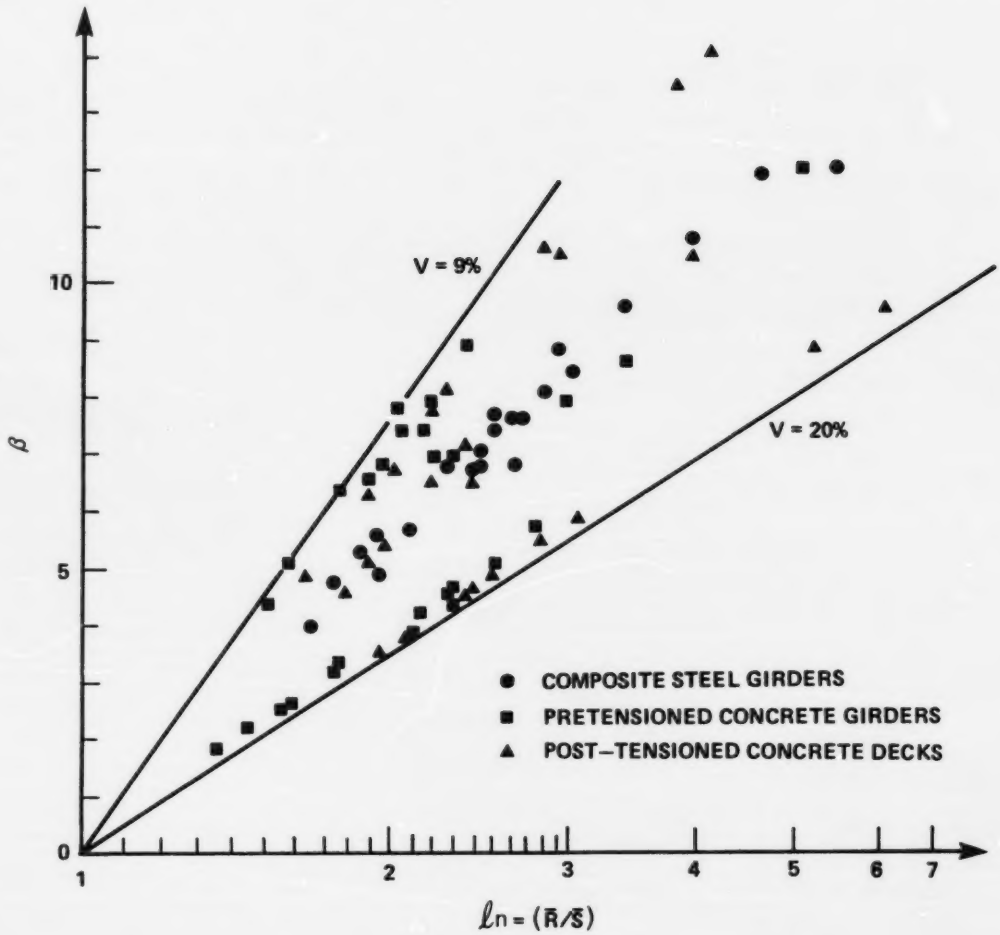

Figure 3/ Safety Indices

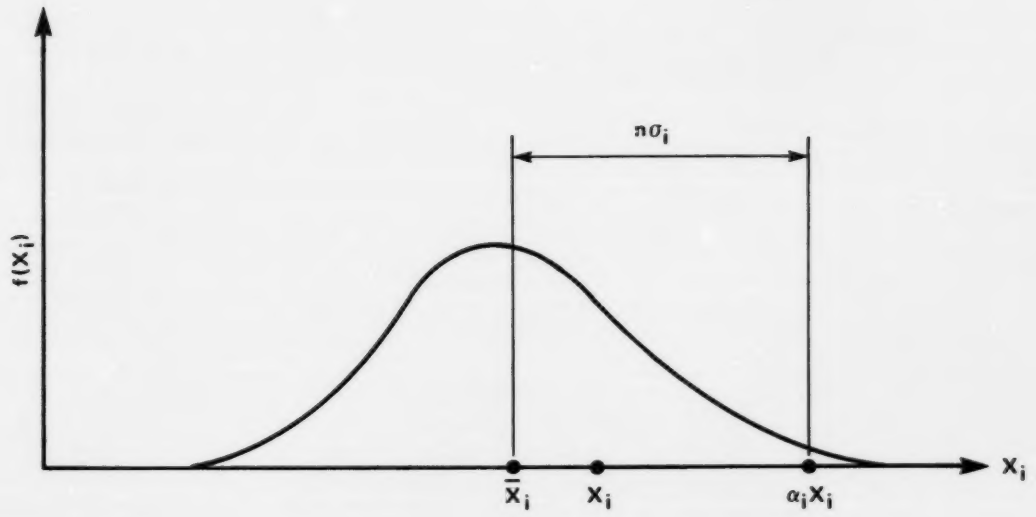

Figure 4/ Probability Distribution of a Load Component

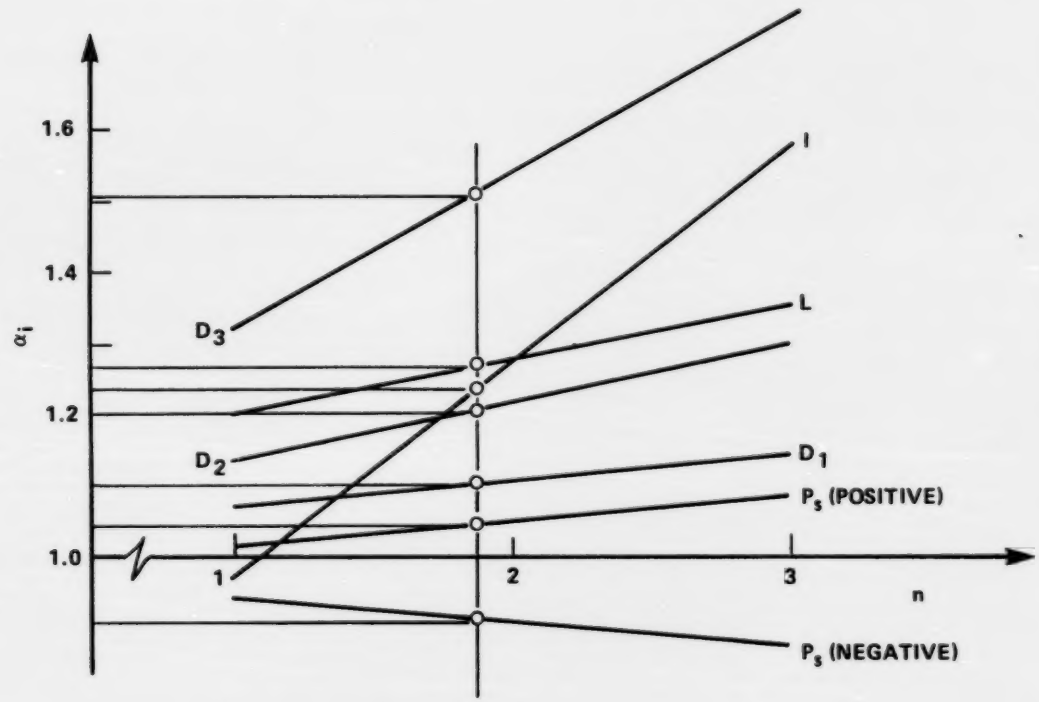

Figure 5/ Preliminary Selection of Load Factors

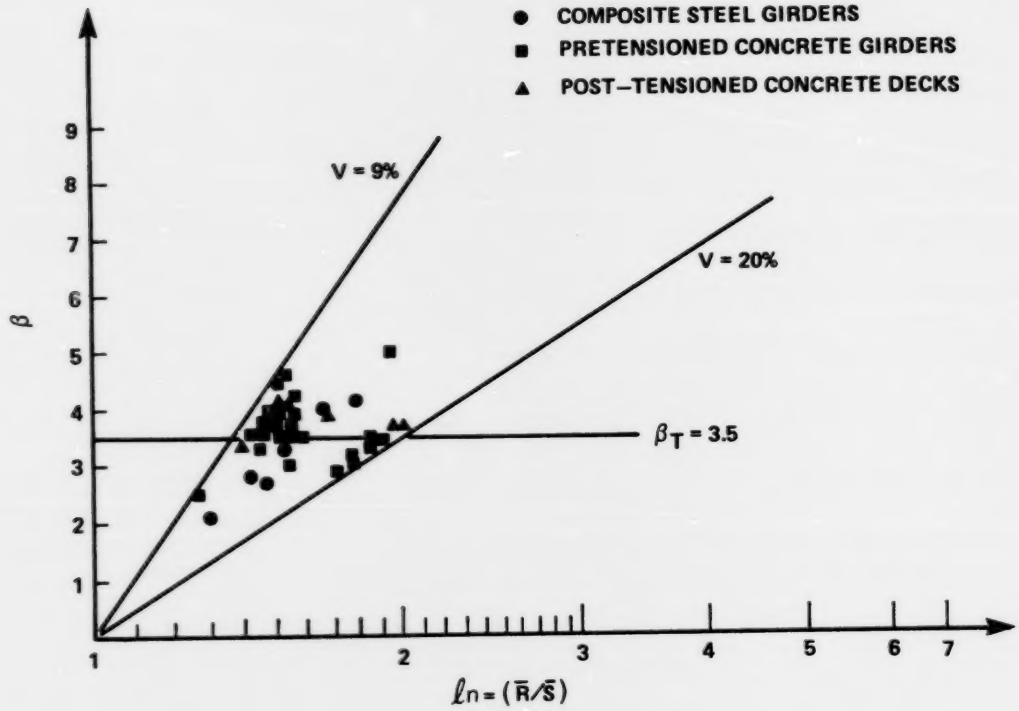

Figure 6/ Safety Indices for the Selected Bridges Designed Using Preliminary Set of Load and Performance Factors

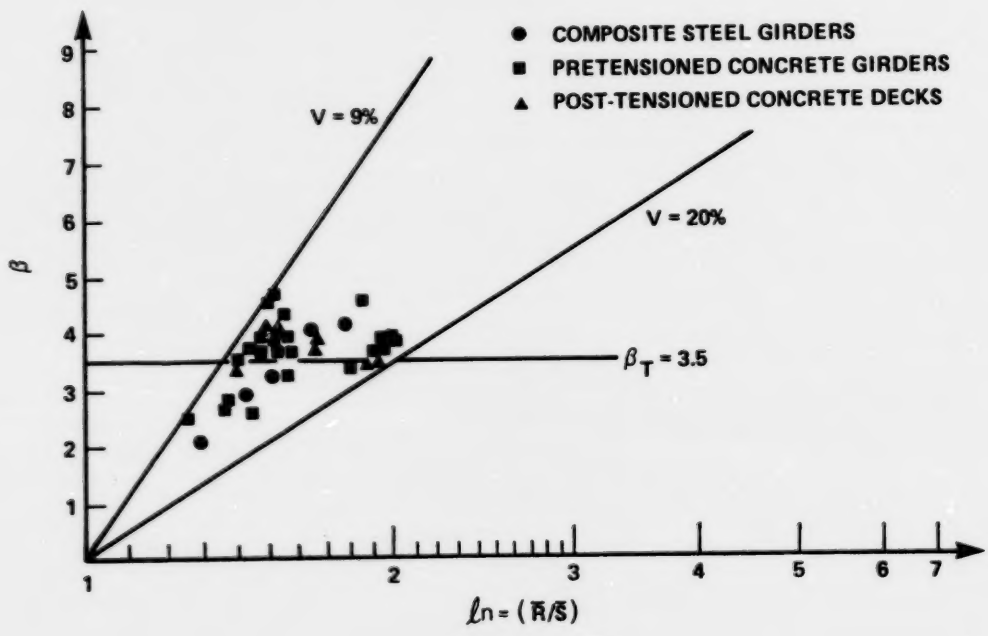

Figure 7/ Safety Indices for the Selected Bridges Designed Using Adjusted Values of Load and Performance Factors

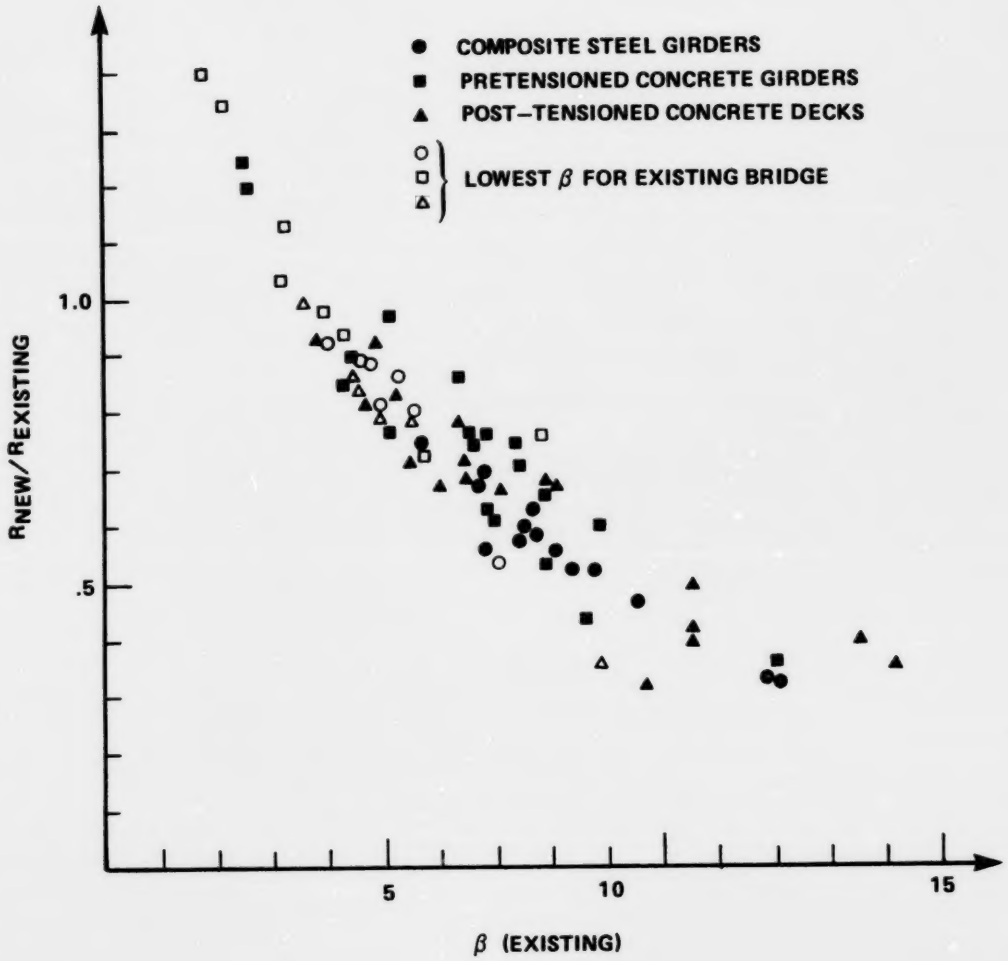

Figure 8/ Comparison of Resistance Required in the Selected Bridges Resulting From the OHBD Code and the Existing Bridge Design Codes